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RESEARCH

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# Investigating the Effect of Bending on the Seismic Performance of Hollow-Core Flooring

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**Abstract**

Even if precast pre-stressed hollow-core (PPHC) slabs are usually designed as simply supported elements, continuity with the supporting beam may exist when constructed together with a reinforced concrete topping and continuity reinforcing bars. During an earthquake (and possibly other lateral load), this continuity may result in bending moments being induced close to the supports as the buildings sway laterally. The response of precast floors to earthquake-induced demands has been addressed by past research. However, further investigation is required to improve understanding of several aspects of precast floor behaviour either revealed or emphasized by recent earthquakes in New Zealand. This paper proposes a mechanics-based modelling approach for the analysis of PPHC slab-to-beam seating connections. The model has been calibrated against existing test data to predict the failure of a PPHC slab under negative bending moments. The numerical outcomes allow comparison of the moment–drift response, principal tensile stresses, and crack progression during loading. The developed modelling approach will allow future studies to exhaustively investigate all aspects of precast floor behaviour by varying the properties and geometry of the PPHC seating connection.

**Keywords** Hollow-core floor, Pre-stress concrete, Precast concrete, Diaphragm behaviour, Finite element method, Fracture mechanics, Numerical modelling

## 1 Introduction

Precast pre-stressed hollow-core (PPHC) floors comprise precast floor units with in situ reinforced concrete topping to form a composite floor system that generally also functions as a diaphragm. Compared with other precast floor options, PPHC floors are distinguished by typically having longer shear spans and/or supporting heavier loads with a reduced self-weight and faster construction time (Brooke et al., 2019).

Diaphragm action (required for the floor to transfer horizontal inertia forces to walls and frames) is obtained by connecting the PPHC units to each other and to the framing beams through cast-in situ reinforced concrete joints. In seismic areas, the upper surface of precast slabs is usually covered with a cast-in situ concrete topping, so as to enhance the strength and stiffness of the floor and its structural performance under lateral loads (Fenwick et al., 2010). Nevertheless, an effective composite action can be achieved only when the topping has adequate thickness, and when proper shear strength is provided at the interface between the topping itself and the slabs (Ueda & Stitmannaithum, 1991).

Fig. 1 shows the typical cross-section of a PPHC floor. The precast units commonly sit on a beam edge with a

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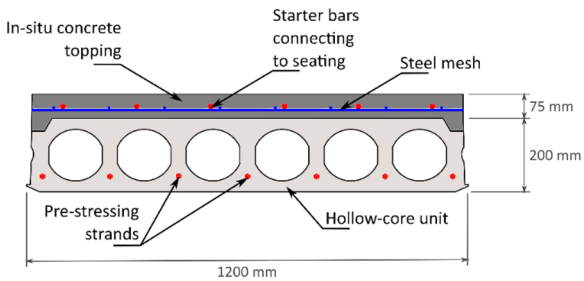


**Fig. 1** Typical hollow-core floor cross-section

cast-in place topping containing passive reinforcement. Additionally, continuity reinforcement or 'starter bars' will be placed to connect the concrete topping to the seating beam. In New Zealand, a wide range of hollow-core sections and seating connection detailing have been adopted. This is because the dimensions and material properties of the precast units have varied over the years as technology has changed and because there was a lack of design guidelines during the 1980s and 1990s, when most buildings with PPHC floors were constructed in New Zealand. It has been found that about 80% of existing buildings with PPHC floors in Wellington have a 65 mm topping slab with welded wire mesh reinforcement, and that over 60% have starter bar lengths not exceeding 600 mm (Puranam et al., 2019). This information is relevant when assessing the likely impact of bending on the floors, as will become evident later in this paper. A side view of the PPHC floor seating detail can be appreciated later in Fig. 7.

PPHC slabs are commonly designed as simply supported members. Nonetheless, the presence of the concrete topping and reinforcement prompts continuity between the units and the supporting structure. Consequently, during an earthquake, significant bending moments can be induced in the PPHC units, close to the supports (Fenwick et al., 2010).

Prior to the 1994 Northridge earthquake, little research had been conducted into the seismic behaviour of precast concrete floor diaphragms. However, noticeable diaphragm flexibility observed in some structures after connections had sustained earthquake damage led engineers to begin questioning the seismic performance of precast concrete floor elements (Corney et al., 2021). Experimental investigations into the behaviour of precast concrete floors and improvements in their performance have been made over the last three decades (Corney et al., 2018; Fenwick et al., 2010; Matthews, 2004; Woods, 2008). Such research has provided a significant basis for understanding the response of precast floors to earthquake-induced demands.

While the details for floor systems with PPHC units have been improved in new buildings, support conditions for units in existing buildings designed before 2006 are likely to lead to significant damage and potentially collapse during design level earthquakes (Brooke et al., 2019). This has been highlighted by the 2010/2011 Canterbury Earthquake Sequence and the 2016 Kaikōura Earthquake in New Zealand. Further investigation is required to improve understanding of several aspects of precast floor behaviour either revealed or emphasized by recent earthquakes or identified prior to the earthquakes but not fully investigated.

The expense of physical testing and the difficulty of tightly controlling the properties of reinforced concrete test specimens makes it impractical to exhaustively investigate all aspects of precast floor behaviour experimentally. Therefore, this paper aims to propose a finite element (FE) approach for evaluating the effect of bending moments on the seismic performance of PPHC floors. For this purpose, a detailed three-dimensional model of PPHC floor seating connections, or PPHC subsystems, has been developed and calibrated against past experimental data by Bueker et al., (2020). The developed model, based on nonlinear fracture mechanics, is then used to study the effect of the starter bar length on the observed failure mode and crack propagation.

## 2 Seismic Response of PPHC Floors

In simply supported pre-stressed concrete members without web reinforcement, such as extruded PPHC units, the shear strength in the high shear regions close to the supports is limited by web-shear cracking (Fig. 2). However, where continuity is established between the PPHC units and the supporting structure, axial tension and bending moments can be introduced into the region near the supports. In this situation, the shear strength is limited by flexure-shear cracking (Fig. 2), which results in the strength being significantly less than the value corresponding to web-shear cracking strength. In this situation, bending moments will develop at the support under gravity and earthquake movement, which induce local displacements and structural actions into the individual floor units, which in turn are likely to cause a brittle shear failure (Woods, 2008). Therefore, the assumption of simple supports can lead to an over-estimate of the shear strength in the presence of continuity actions, as the strength cannot be based on analytical calculations of simply supported members alone.

Negative moment failure (NMF) refers to the failure of a PPHC floor due to the exceedance of the tensile strength at the top section, followed by propagation of cracking through the depth of the unit.

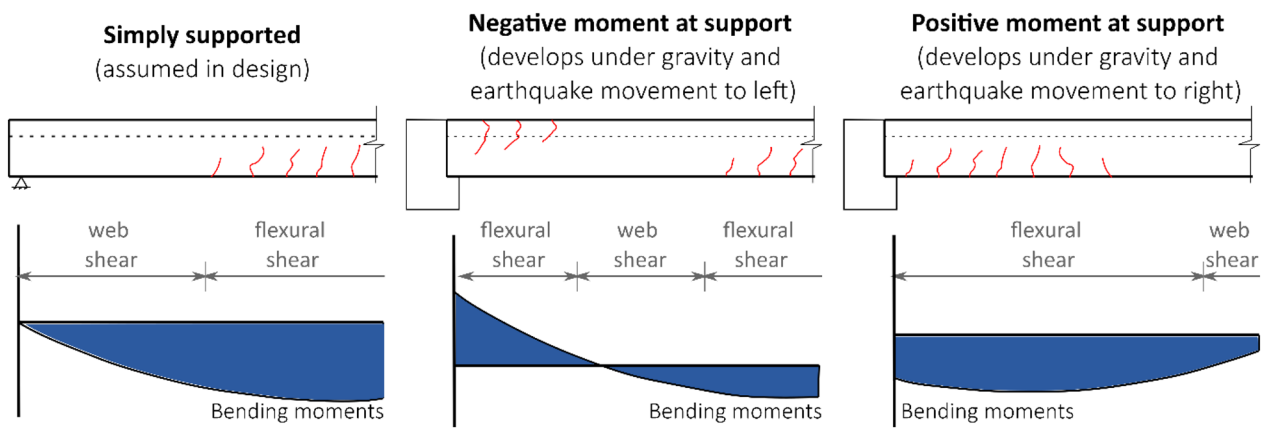


**Fig. 2** Flexural and web-shear limits on shear strength of concrete [adapted from Fenwick et al. (2010)]

Such failure typically manifests itself at the end of the starter bars, which are provided to connect the floors to the supporting structure. The occurrence of a negative moment failure at the end of the starter bars is dependent on a number of factors that affect the magnitude of the moment that occurs at that location, including the strength of starter bars and the possible presence of other additional sources of strength (Fenwick et al., 2010). The most vulnerable cases are those with starter bars terminated too close to the support, as there is a drop in negative moment capacity at the end of the starter bars, which results in a weak section. After a crack forms in the topping it can then propagate through the depth of the section, into the webs of the unit, and then horizontally at the bottom flange, as shown in Fig. 3a (Woods, 2008). This causes loss of gravity load-carrying capacity and potential collapse of the floor unit (Fig. 3b). In contrast, positive moment failure (PMF) may happen upon reverse loading, when the starter bars are not terminated too close to the support and the beam–floor interface is sufficient to transfer positive moments into the floor unit (Fig. 2).

## 3 Finite Element Modelling Approach

### 3.1 Proposed Numerical Approach

To provide an improved understanding of the effect of bending moments on the seismic behaviour of PPHC floors, an FE modelling approach is proposed in this work. The detailed three-dimensional FE model has been developed based on nonlinear fracture mechanics using the software Midas FEA (MIDAS Information Technology, 2016). The model is calibrated against existing past experimental results from experimental testing carried out by Bueker et al., (2020) on simplified hollow-core seating connections to capture the NMF cracking mechanism.

The portion of the diaphragm analysed in this work is highlighted in Fig. 4, which consists of a single unit within a floor area of a concrete frame building. The half-span of the slab plus the connection to the seating beam is herein referred to as a PPHC floor sub-system.

Fig. 5 shows the geometry and mesh of the FE model developed. The concrete (used for the PPHC unit, in situ topping and seating beam) has been modelled using solid six-node brick elements. The complex geometry of the

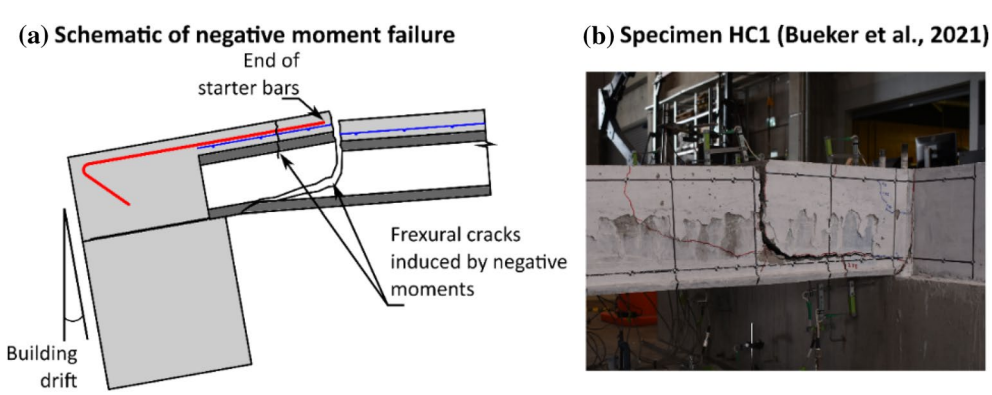


**Fig. 3** Negative and positive flexural failure in a hollow-core floor system [adapted from Fenwick et al. (2010)]

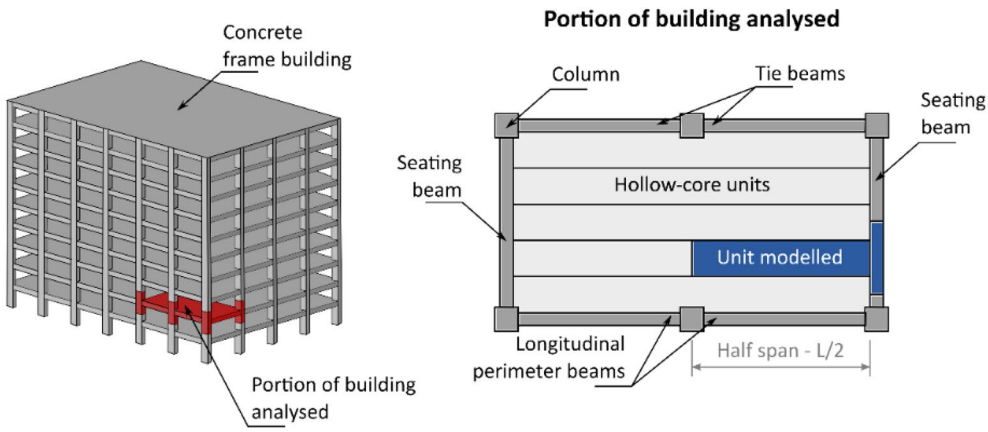


**Fig. 4** PPHC sub-system origin [adapted from Jensen (2007)]

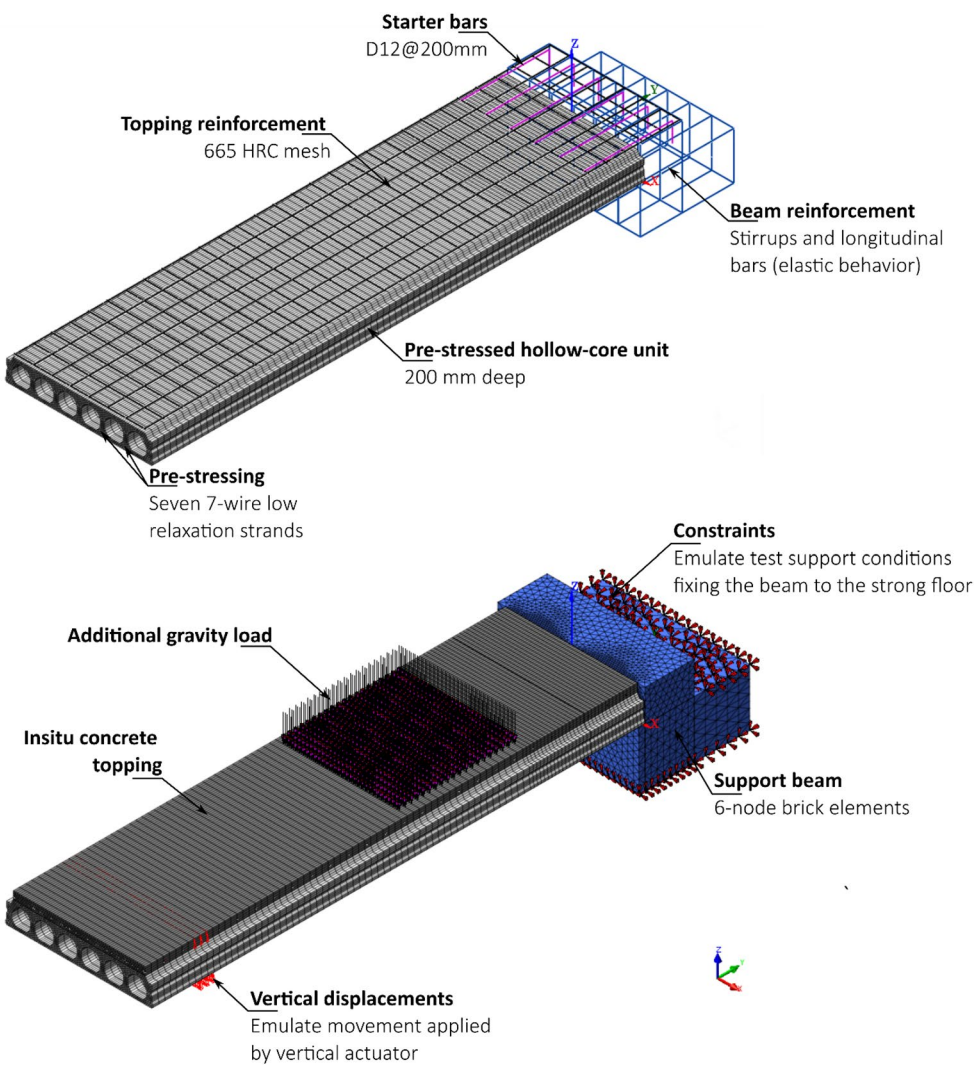


**Fig. 5** PPHC seating connection FE model

sub-system has been modelled via the following steps: (i) box elements were created to represent the geometry of the PPHC unit, topping and beam; (ii) neighbouring surfaces were joined such that the elements will deform together when loaded; (iii) the box elements were discretized into six-node mesh elements assuring that the nodes shared between surfaces are aligned with each other, or shared.

The creation of the mesh for the PPHC unit demands particular attention. First, the cross-section of the PPHC unit was discretized into three-node plane mesh elements, and then, the two-dimensional mesh was extruded along the length of the unit, resulting in six-node brick elements. The use of four-node plane mesh elements, which extruded result into an eight-node brick element, has been considered. However, due to the complex cross-section geometry of the hollow-core units, the four-node mesh elements resulted in elements that differ greatly in area. Therefore, the authors opted for three-node plane elements and six-node brick elements, which result in elements with more consistent dimensions.

The sub-system model shown includes a total of 235,000 elements, approximately. These elements comprise roughly 225,000 brick elements involving about 117,000 brick elements composing the PPHC slab, 58,000 the concrete topping, and 55,000 the seating beam. There are about 4100 embedded line elements used to define the reinforcement bars and the pre-stressing strands, and about 400 interface elements. The number of elements will appreciably vary if the span of the PPHC floor considered is longer or the geometry of the PPHC unit or thickness of the concrete topping varies.

The properties of the extruded hollow-core concrete have been adopted from the modelling recommendations by Sarkis et al., (2022b) based on a detailed sensitivity analysis and concrete material characterization testing (Sarkis et al., 2022a). The rotating total strain crack model has been adopted (Selby & Vecchio, 1993; Vecchio & Collins, 1986) to capture the concrete failure mechanism. The rotating crack model is a method in which the directions of the cracks are assumed to continuously rotate depending on the changes in the axes of the principal strains. The algorithm for the rotating crack model is relatively simple when compared to the alternative of a fixed model, as so, convergence is superior since this model is unrelated to the previous cracking conditions. The total strain crack model materializes the softening function of the concrete based on fracture energy. In this case, the stress–strain relationship by Cornelissen et al., (1986) was selected to represent the tensile behaviour of the concrete. The compressive behaviour has been defined following the stress–strain relationship proposed by Thorenfeldt (1987). The behaviour of the cast-in situ concrete of the seating beam and the topping concrete was assumed to be nonlinear, and hence, the same constitutive models were selected. For simplification purposes, the possibility of considering the behaviour of the concrete of the seating beam elastic was studied as an alternative. Nonetheless, it affected the modelling results as it did not allow for any cracking at the seating section and at the back of the unit.

Steel mesh was used as topping reinforcement, and the transverse and longitudinal reinforcement of the beam and the pre-stressing strands were modelled as embedded line elements. The topping reinforcing mesh and the starter bars were modelled following the Von Mises yielding criterion, whereas all the beam reinforcement was assumed to remain elastic. A perfect bond between the topping and beam reinforcement and the concrete was assumed for simplification.

The interaction between the pre-stressing strands and the concrete has been represented by a parabolic pre-stress distribution, as per Yang, (1994). The pre-stress losses and transfer length of the strands have been estimated following the New Zealand concrete standard (NZS3101, 2006), and the end-slip of the strands according to Brooks et al., (1988). No interface elements were introduced to describe the strands–concrete interaction, as this has been indirectly accounted for through the equivalent pre-stress distribution.

Interface elements are used to study the interface movements at the boundaries between materials, in this case, the crack surface between the two concrete materials. These elements analyse the crack surface by relating the forces acting on the interface to the relative displacement of the two sides of the interface. A layer of interface elements was placed between the back face of the PPHC unit and the face of the beam. The interface is defined by using a general finite element formulation, with the thickness of the elements assumed to be zero, thus keeping model geometry unaltered. The softening model proposed by Hordijk (Cornelissen et al., 1986; Hordijk, 1992) is used to describe the nonlinear behaviour of the structural interface element.

Vertical displacements have been applied at the cantilever end of the PPHC slab (Fig. 5). Positive and negative displacements in the $Z$-axis are alternated by using construction stages in the nonlinear analysis. This cyclic loading aims to induce rotations at the beam–floor joint, simulating the effect of the building's lateral sway caused by seismic actions. For the nonlinear analysis, through the construction stages, the energy-controlled Newton–Raphson iteration scheme was employed, with an energy norm of 0.005 and a loading rate of 0.17 mm/step. Restraints in three translational directions were placed at the top and bottom of the beam, as shown in Fig. 5,

to mimic the test support conditions fixing the beam to the strong floor of the laboratory. Lastly, the additional gravity load applied to the floor has been considered and modelled as a uniformly distributed pressure load acting on the top of the topping.

### 3.2 Calibration of the FE Model

The model presented herein followed a two-way validation process. The PPHC model was first calibrated against material testing to identify the properties defining the behaviour of the extruded concrete. The model was then initially validated against experimental data (Pajari, 2004a, 2004b; Sarkis et al., 2022a, 2022b) to predict brittle failure mechanisms due to shear and torsional actions. The work presented herein aims to advance the previously validated FE model of PPHC units into a more complex sub-system model capable of predicting flexural moment failure mechanism.

To calibrate the sub-system FE model, a test carried out by Bueker et al., (2020), which presented NMF, was selected. The specimen consisted of a 200 mm deep PPHC unit, which is the most commonly found unit depth in existing buildings in New Zealand (Puranam et al., 2019). Fig. 6 shows the general set-up employed during the sub-system testing. To create the PPHC sub-system, a single flooring unit is seated on one end at a section of beam. The seating beam was fixed to the laboratory's strong floor through post-tensioned rods. A vertical actuator, with displacement control, was positioned at the cantilever end of the slab. The actuator was able to simulate relative movements between the PPHC floor and the seating beam by imposing rotations on the beam–floor connection.

The specimen was instrumented to measure the applied load, displacements and crack widths. Actions transferred to the spreader beam and to the specimen were monitored using a load cell at the shaft of the actuator. Additionally, a grid of displacement gauges was placed on one of the faces of the PPHC unit to measure crack widths near the supporting beam, where failure was expected to happen.

Before the loading started, a wood dunnage was placed at the bottom of the slab, providing support to the PPHC slab and avoiding pre-testing deformations. An additional gravity load of 500 kg was placed at 1450 mm from the seating end of the PPHC unit.

Table 1 summarizes the main features of the tested specimen, including the geometry of the specimen and the compressive strength of the beam and topping concrete. The detail of the floor seating connection is shown in Fig. 7. The PPHC unit had a length of 4000 mm, with the actuator located 500 mm away from the cantilever end of the unit. The 50 mm seating had a mortar pad, made of Sika 212 grout, as a bearing surface. The measured compressive strength of the concrete was 40 and 26 MPa for the seating beam and concrete topping, respectively. The starter bars had a diameter of 12 mm and extended 400 mm away from the beam–slab interface. The reinforcing welded steel mesh extended all the way to the top of the beam and had a typical yielding strength of 500 MPa.

Fig. 8a shows the loading protocol employed during the experimental testing by Bueker et al., (2020) with the black dashed line. The model proved to be quite sensitive to the loading history. The first half cycle of the loading protocol (with the solid red line) corresponds to the

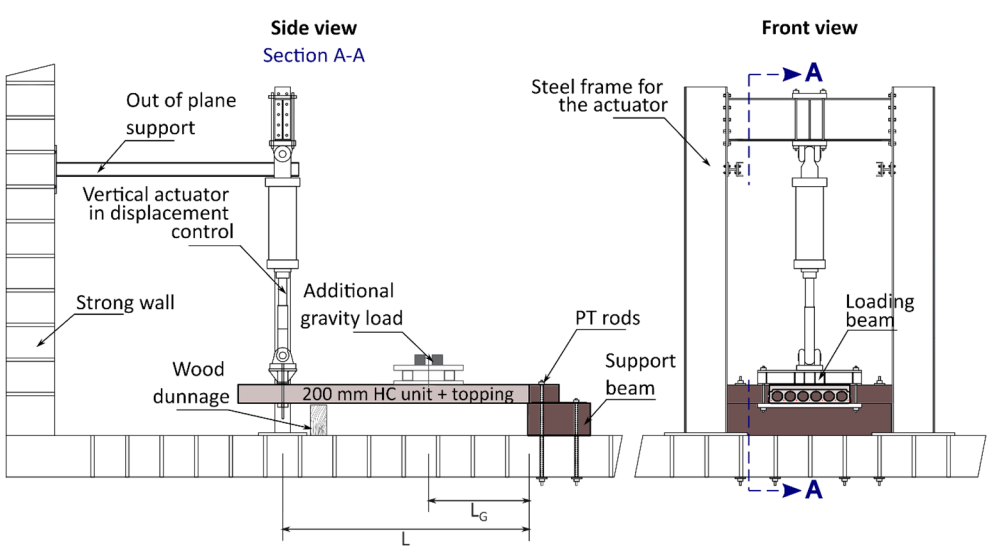


**Fig. 6** Sub-assembly experimental test set-up

**Table 1** Summary of key features of the sub-system test used for the calibration of the FE model

| Feature | Value |
|---|---|
| Observed failure mode | NMF |
| Length of the HC unit, $L_{HC}$ (mm) | 4000 |
| Distance from the seating end of the HC to the applied load, L (mm) | 3500 |
| Distance for the seating end of the HC to the additional gravity load, $L_G$ (mm) | 1450 |
| Seating length, $L_s$ (mm) | 50 |
| Bearing surface on RC ledge | Sika 212 grout |
| Seating beam compressive strength, $f_{c,beam}$ (MPa) | 40 |
| Concrete topping compressive strength, $f_{c,top}$ (MPa) | 26 |

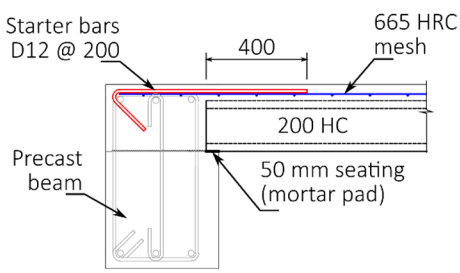


**Fig. 7** Connection detailing of the sub-system test by Bueker et al. (2020)

lifting of the slab, to about 0.1% drift and back to zero, to remove the wooden dunnage from underneath the slab. Fig. 8b shows the moment–drift response of the tested specimen. During the removal of the wooden dunnage, the specimen remained in the elastic range. Then, through the first half cycle down to − 1.0% drift, strength degradation took place as the flexural cracks evolved into a fully developed negative moment crack. Additional cycles contributed to widening this crack until collapse. For the calibration of the FE model in this work, only the first loading cycle is examined, as most of the cracking first appeared before − 1.0% drift. The section of the loading protocol considered in the numerical analysis is highlighted in Fig. 8(a).

## 4 Results and Discussion

### 4.1 Negative Moment Failure

Fig. 9 compares the experimental moment–drift relationship with the FE prediction. The moment has been estimated at the beam–slab interface and the drift according to the displacements measured at the location of the applied load. At the start of the negative cycle, the negative moment at the interface increases almost linearly, as the drift induced by the applied load increases. A good correlation is seen between the model and experiment in the stiffness of the specimen before the maximum negative moment is reached, or initial stiffness. The negative flexural crack first formed at the end of the starter bars, which is shown in the moment–drift plot as a drop in the moment capacity (marked with point A in Fig. 9). After the formation of the first negative flexural crack, both curves, numerical and experimental, showed significant stiffness degradation. The maximum moment

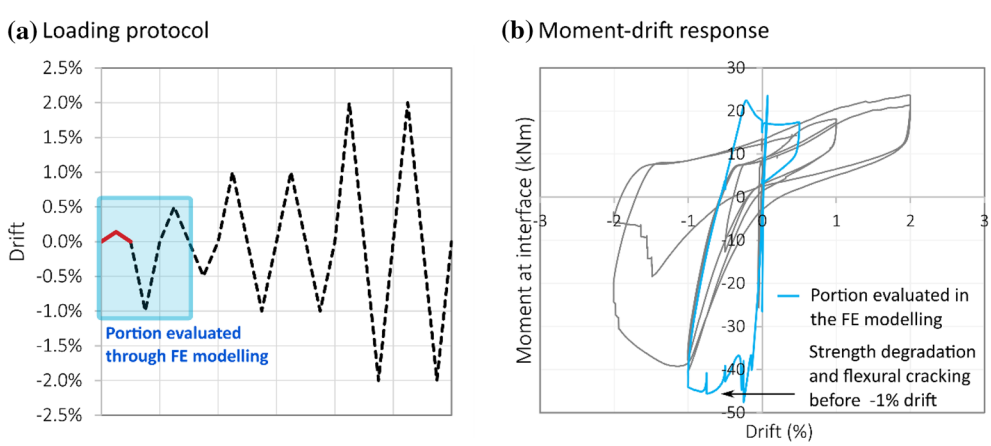


**Fig. 8** Connection tested for NMF.

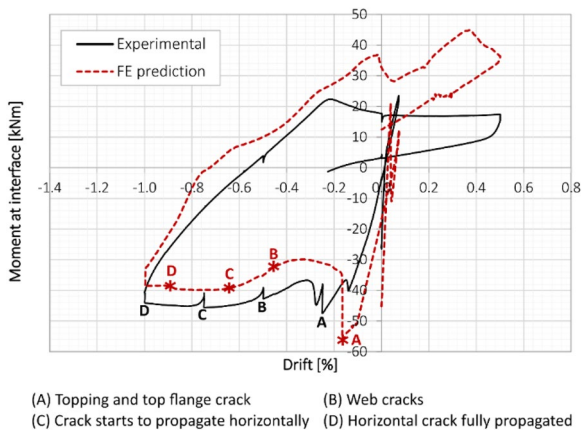


**Fig. 9** Comparison of the numerical and experimental drift versus moment at the beam–unit interface

obtained during the test was 47.6 kNm at 0.25% drift. The FE model predicted a maximum moment of 56.1 kNm at 0.16%, meaning a moment 16% larger than the maximum moment obtained experimentally.

Figs. 10 and 11 compare the cracking observed during the test with the FE-predicted crack pattern and principal tensile stresses, respectively. The same damage progression is marked in the plot, shown in Fig. 9 with the points A to D, and the corresponding moment and drift values are listed in Table 2. After the negative flexural cracks first formed at the top of the slab, at the end of the starter bars (point A), the flexural crack extends vertically towards the bottom of the unit as the web cracks (point B). This took place in the test approximately at a drift of 0.5% and a moment of 40.3 kNm, whereas the FE model predicted a moment of 33.2 kNm at 0.46% drift. Subsequently, the crack starts to propagate horizontally at the top of the bottom flange (as shown with point C) at a drift of 0.75% and 0.65%, and a moment of 45.7 kNm and 39.4 kNm for the test and the FE prediction, respectively. Finally, the horizontal crack fully propagates in both directions (point D) forming an inverted 'T' shape. This final step of the damage progression was reached in both cases before 1% drift, and the moment measured during the test at this drift was 44.2 kNm, while the FE predicted a moment of 38.4 kNm. The results suggest that the proposed numerical approach represents a rational technique for analysing cracking development and propagation in PPHC sub-systems subjected to negative bending moments.

During the early stages of the development of the FE model, the first section of the loading protocol corresponding to the removal of the dunnage (Fig. 8) was not considered, as it corresponds to displacements under 3 mm, and since the specimen behaved elastic, with no noticeable cracking. The resulting model predicted the cracking development rationally at drift levels closely comparable to the experimental results. Nonetheless, the initial stiffness of the sub-system and the maximum negative moment capacity were greatly overpredicted. Once those initial displacements were added, the stiffness and moment capacity prediction improved significantly. This proved the sensitivity of the moment predictions to small rotations induced at the beam–slab interface and also

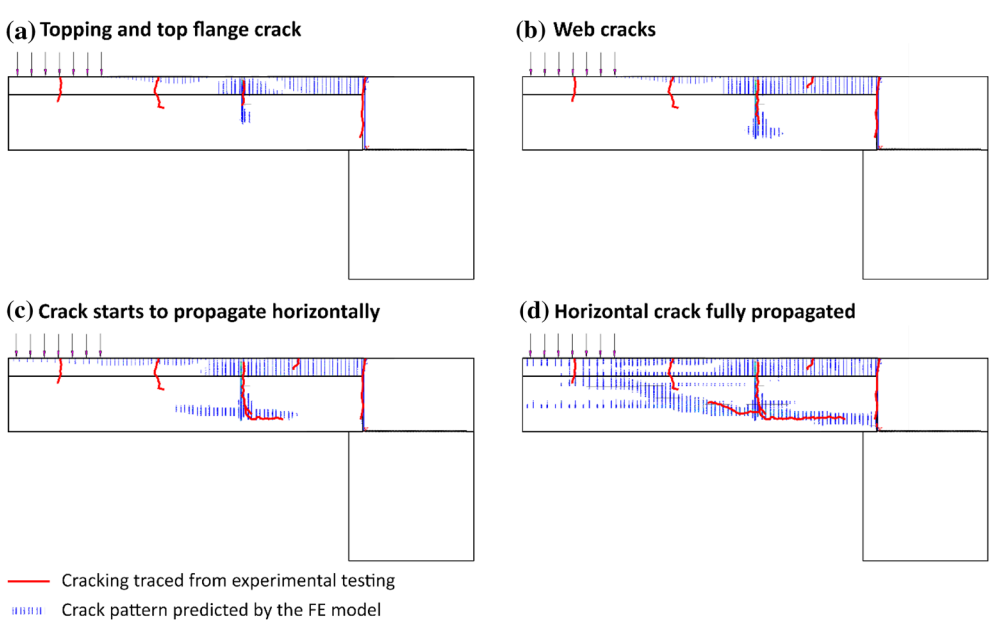


**Fig. 10** Comparison of the crack pattern predicted by the FE model and the cracks traced from the experimental testing

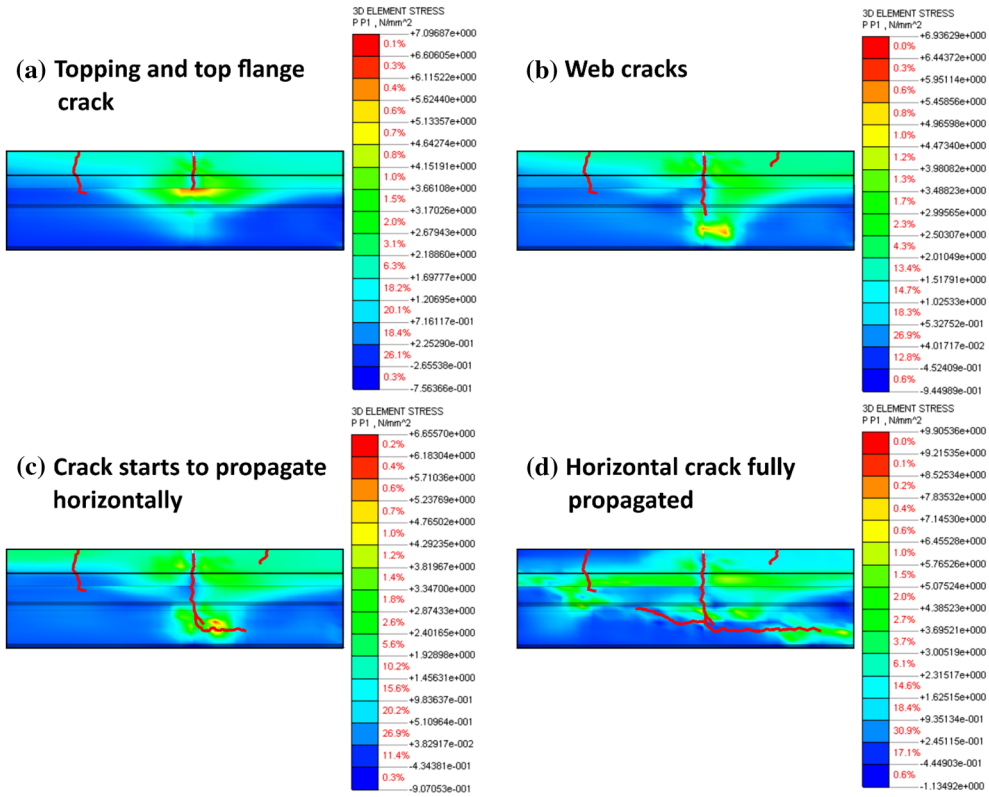


**Fig. 11** Progression of principal tensile stresses in the slab during negative loading, up to − 1.0% drift

**Table 2** Negative moment and associated drift values at different instants of the damage progression

| Damage snapshot | Description | Experimental results | | FE prediction | |
|---|---|---|---|---|---|
| | | Moment | Drift | Moment | Drift |
| | | kNm | % | kNm | % |
| A | Topping and top flange crack | 47.6 | 0.25 | 56.1 | 0.16 |
| B | Web cracks | 40.3 | 0.50 | 33.2 | 0.46 |
| C | Crack starts to propagate horizontally | 45.7 | 0.75 | 39.4 | 0.65 |
| D | Horizontal crack fully propagated | 44.2 | 1.00 | 38.4 | 0.90 |

suggests that the actual behaviour of precast floor slabs may be quite sensitive to loading characteristics, which in turn suggests there may be significant uncertainty in the seismic capacity of PPHC floor units. Future attempts to predict the capacity of PPHC floors should account for the sensitivity of the floors to imposed deformations and the fact that failure mechanisms present themselves in a brittle manner at early displacement cycles.

### 4.2 Effect of the Length of Starter Bars

NMF in PPHC floors typically happens when the starter bars are terminated too close to the support, which in principle can be remediated by placing longer starter bars. Hence, with the purpose to investigate the effect of employing longer starter bars in the seismic response of PPHC floors, the sub-system described in Sect. 3.2 has hereinafter been modelled with 600-mm-long bars.

Fig. 12a illustrates a PMF damage mechanism on a PPHC floor. At the end of the PPHC units, the pre-stressing strands will not be fully developed and therefore, the unit will be only capable of resisting a very small percentage of their design strength as the positive moment flexural strength will depend predominantly on the tensile strength of the concrete. For this reason, flexural cracking at this location is generally initiated at low drift levels, and any additional deformation is typically concentrated

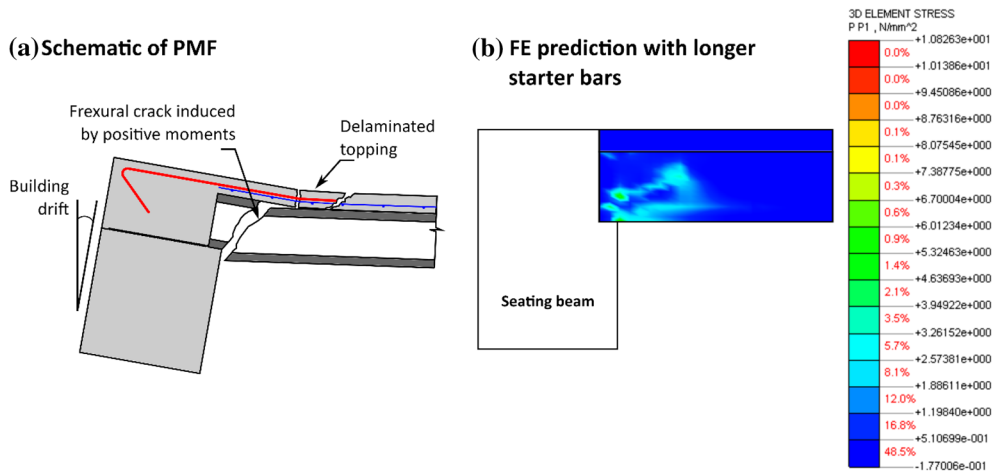


**Fig. 12** PMF schematic and principal tensile stresses predicted by the FE model with longer starter bars

in the cracked section (Fenwick et al., 2010). Once a positive crack has formed, it creates a weak section, which widens when axial tension is applied to the floor from the elongation of the beams parallel to the PPHC units. As the crack width increases, gravity load transfer is dependent on a dowel action of the strand. At this point, strand slip is expected to result in the collapse of the floor unit (Woods, 2008).

By extending the length of the continuity reinforcement, the capacity of the connection increases, and sufficient positive moment is transferred to the connection, so that instead of triggering a NMF at a drift of 0.3%, the system can reach a drift of 0.5% in the second positive cycle before a PMF occurs. The drift capacity in the (non-critical) negative loading direction is also shown to increase by avoiding the NMF, but to a lesser extent. Fig. 12b presents the principal tensile stress distribution obtained numerically at the moment of failure. A good correlation between the theory and the modelling results is shown, which demonstrates further the effectiveness of the proposed FE approach. Moreover, this example illustrates the potential value of this modelling and analysis approach in gauging the impact of retrofit efforts (in this case, the installation of additional reinforcement in the topping slab to avoid NMF). The FE modelling approach developed to date should permit future studies to exhaustively investigate all aspects of precast floor behaviour by varying the properties and geometry of PPHC seating connections.

## 5 Conclusions

The present work investigated the seismic performance of PPHC floors under bending moments through FE modelling. The research met its aim of providing a numerical approach to provide insight into the likely seismic performance of PPHC slab-to-beam seating connections. This work also illustrates the potential value of the FE modelling and analysis approach in gauging the impact of retrofit efforts for precast hollow-core flooring systems.

From the results obtained, the following conclusions can be drawn:

- When rotations were induced in the PPHC connection, cracks appeared at the end of the starter bars, at the top of the slab, and then propagated vertically down the webs of the hollow-core unit before extending horizontally at the top of the bottom flange of the unit, forming a full NMF mechanism at less than 1% drift.
- The finite element modelling approach developed has a tendency to overpredict the initial stiffness of the connection and the moment at which the NMF cracking initiates, but proved to be effective at predicting the crack propagation and failure mechanism at satisfactory drift levels.
- The initial stiffness of the PPHC sub-system appeared to be highly affected by the loading protocol and in particular, a small rotation demand that was imposed early in the test. This showed that it is necessary to include any small displacements undergone, for example, lifting of the slab to remove the wooden dunnage before the start of the test. Furthermore, this finding suggests that the actual behaviour of precast floor slabs may be quite sensitive to loading characteristics, which in turn suggests there may be significant uncertainty in the seismic capacity of PPHC floor units prone to NMF.
- It is recommended for future experimental efforts, aimed at the calibration of similar sub-system models, to pay particular attention to the instrumenta-

tion and recording of all aspects of the specimen construction, manipulation, and testing. The PPHC sub-systems are highly sensitive to imposed deformations, and so, exhaustive data recording is necessary for any further modelling attempts.

- By re-running the FE analyses with a longer starter bar length, it was demonstrated that the length of the starter bars affected the anticipated failure mode, as was expected. The use of shorter starter bars triggered NMF at a drift of − 0.9%, in line with the experimental test used as a reference, whereas the longer starter bars avoided NMF and saw the development of a PMF at a drift of 0.3% (on the second positive cycle to 0.5% drift). This work also illustrates the potential value of the finite element modelling and analysis approach in gauging the impact of retrofit efforts for precast hollow-core flooring systems.

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**Author contributions**
The calculations, modelling, analysis and writing of the research presented in this paper has been undertaken by the corresponding authors AIS. All authors contributed to the conceptual development and research methodology. And finally, revision of the final version of the manuscript submitted was carried out by the co-authors TRS, RN and EB. All authors read and approved the final manuscript.

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**Availability of data and materials**
The datasets used and/or analysed during the current study are available from the corresponding author on reasonable request.

## Declarations

**Competing interests**
The authors declare that they have no competing interests.

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